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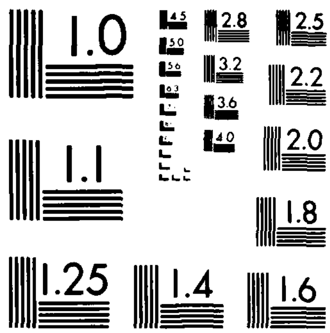

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MILITARY HYDROLOGY

Report 12

CASE STUDY EVALUATION OF ALTERNATIVE DAM-BREACH FLOOD WAVE MODELS

Volume I: Main Report

by

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This report provides a comparative evaluation of several leading state-of-the-art models for dam-breach flood forecasting. Nine alternative models were applied using data sets from four case studies. Results obtained were compared between models and with available measured data. Model accuracy, versatility, and ease-of-use were evaluated, and complexities and weaknesses were identified. The sensitivity of model results to various input data parameters was also investigated. The quantitative results documented by the report provide an

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empirical data base for analyzing the performance of the models under various conditions.

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PREFACE

The work reported herein was conducted under Department of the Army Project No. 4A762719AT40, "Mobility and Weapon Effects Technology," Task Area BO, "AirLand Battlefield Environment," Mission Area, "Combat Support," Work Unit 052, "Induced Floods as Linear/Area Obstacles," under the auspices of the Battlefield Terrain Working Group of the AirLand Battlefield Environment Thrust. The study was sponsored by the Office, Chief of Engineers (OCE). Dr. Clemens A. Meyer was the OCE Technical Monitor.

The study was conducted by the US Army Engineer Waterways Experiment Station (WES) under the general supervision of Dr. John Harrison, Chief of the Environmental Laboratory, and Dr. Lewis E. Link, Chief of the Environmental Systems Division, and under the direct supervision of Mr. M. P. Keown, Chief of the Environmental Constraints Group (ECG), and Mr. J. G. Collins, ECG. Mr. M. R. Jourdan, ECG, Principal Investigator, Work Unit 052, provided technical assistance and review. This report was prepared by Dr. Ralph A. Wurbs, who is an Assistant Professor at Texas A&M University working under an Intergovernmental Personnel Act agreement as a Research Engineer, ECG. The report was edited by Ms. Jessica S. Ruff of the WES Publications and Graphic Arts Division.

COL Allen F. Grum, USA, was the previous Director of WES. COL Dwayne G. Lee, CE, is the present Commander and Director. Dr. Robert W. Whalin is Technical Director.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4,046.873	square metres
acre-feet	1,233.489	cubic metres
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
inches	2.54	centimetres
square miles	2.589998	square kilometres

MILITARY HYDROLOGY

CASE STUDY EVALUATION OF ALTERNATIVE DAM-BREACH FLOOD WAVE MODELS

Main Report

PART I: INTRODUCTION

Background

1. Under the Meteorological/Environmental Plan for Action, Phase II, approved for implementation on 26 January 1983, the US Army Corps of Engineers (USACE) has been tasked to implement a Research, Development, Testing, and Evaluation program that will: (a) provide the Army with environmental effects information needed to operate in a realistic battlefield environment, and (b) provide the Army with the capability for near-real time environmental effects assessment on military material and operations in combat. In response to this tasking, the Directorate for Research and Development, USACE, initiated the AirLand Battlefield Environment (ALBE) Thrust program. This new initiative will develop the technologies to provide the field Army with the operational capability to perform and exploit battlefield effects assessments for tactical advantage.

2. Military Hydrology, one facet of the ALBE Thrust, is a specialized field of study that deals with the effects of surface and subsurface water on planning and conducting military operations. In 1977, the Office, Chief of Engineers, approved a military hydrology research program. Management responsibility was subsequently assigned to the Environmental Laboratory, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

3. The objective of military hydrology research is to develop an improved hydrologic capability for the Armed Forces with emphasis on applications in the tactical environment. To meet this overall objective, research is being conducted in four areas: (a) weather-hydrology interactions, (b) state of the ground, (c) streamflow, and (d) water supply.

4. Previously published Military Hydrology reports are listed inside the front cover. This report is the third which contributes to the streamflow modeling area. Streamflow modeling is oriented toward the development of procedures for rapidly forecasting streamflow parameters including discharge,

velocity, depth, width, and flooded area from natural and man-induced hydrologic events. Specific work efforts include: (a) the development of simple and objective streamflow forecasting procedures suitable for Army Terrain Team use, (b) the adaptation of procedures to automatic data processing equipment available to Terrain Teams, (c) the development of procedures for accessing and processing information included in digital terrain databases, and (d) the development of streamflow analysis and display concepts.

Purpose and Scope

5. The study reported here was conducted under the "Induced Floods as Linear/Area Obstacles" Work Unit of the Department of Army Project "Mobility and Weapon Effects Technology." The objective of the work unit is to provide the Armed Forces improved capabilities for forecasting the downstream flood flow impacts resulting from controlled or uncontrolled (dam breach) releases from single or multiple dams. The objective of the investigation reported here was to develop a basis for evaluating and comparing selected alternative dam breach flood wave models by applying the models to actual field data.

6. The work unit was initiated with a comprehensive literature survey regarding dam breach flood wave modeling supplemented by discussions with a number of model developers and users (Wurbs 1985). Based on this review, several leading models representative of the current state-of-the-art were selected for detailed study. The present study consisted of applying the models to several case study data sets. Results obtained with the alternative models were compared between models and with available measured data. Model accuracy, versatility, and ease-of-use were evaluated and complexities and weaknesses identified. The sensitivity of model results to various input data parameters was also investigated. The results obtained, experience gained, and lessons learned from the case study analyses significantly contribute toward providing a sound basis for selecting and adapting models for military use. The quantitative results summarized in this report provide an empirical data base for analyzing the performance of the models under various conditions.

Study and Report Organization

7. The study involved application of nine alternative dam breach flood wave models using data sets from four case studies. However, all of the data sets were not analyzed with all of the models. Table 1 is a matrix indicating which data sets were analyzed with each model.

8. This report consists of four volumes. Volume I is a main report which addresses the overall study and summarizes the results of the several case studies. Volumes II, III, and IV provide the detailed documentation of the Teton (original data), hypothetical prismatic channel, and Laurel Run case studies, respectively. The models and case studies are described in the following sections of this report.

9. The Stillhouse Hollow case study and part of the Teton case study were performed by Captain David N. Buttery in partial fulfillment of the requirements for the degree of Master of Engineering. Captain Buttery is an army officer who, in May 1984, completed a 21-month educational assignment as a graduate student in the Civil Engineering Department at Texas A&M University. Captain Buttery completed the technical project component of his academic program by working with Dr. Wurbs on this study. He documented his work in a separate report (Buttery 1984). Although the detailed documentation of results is not repeated here, his work is incorporated in the summary discussions of this main report.

Table 1

Models Applied in Each Case Study

Model	Case Study: Laurel Run	Hypothetical Prismatic	Teton Original Data	Teton Test Data	Stillhouse Hollow
NWS Dam-Break Flood Forecasting Model (DAMBRK)	X	X	X		X
SWD Flow Simulation Model (FLOW SIM 1)	X	X	X		X
SWD Flow Simulation Model (FLOW SIM 2)	X	X	X		X
HEC Flood Hydrograph Package (HEC-1)	X	X	X	X	X
NWS Simplified Dam-Break Model (SMPDBK)	X	X	X	X	X
HEC Dimensionless Graphs Procedure	X	X		X	X
SCS Simplified Dam-Breach Routing (TR 66)		X		X	X
Military Hydrology Bulletins 9 and 10				X	X
DIA Outflow Hydrograph Computation Method			X		

Note: Volume I addresses all of the case studies from an overview perspective. The Teton (original data), hypothetical prismatic channel, and Laurel Run case studies are documented in detail by volumes II, III, and IV of this report. The Stillhouse Hollow and Teton (test data) case studies are documented in detail by Buttery (1984).

PART II: CASE STUDIES

10. The dam breach flood wave models were tested and compared by application to the following case studies: (1) Teton Dam, (2) hypothetical prismatic channel, (3) Laurel Run Dam, and (4) Stillhouse Hollow Dam. The Laurel Run and Teton case studies involved field data sets from actual dam failures. The hypothetical prismatic channel case study used the Teton reservoir and dam data but replaced the complex Teton Valley geometry with a prismatic channel. Stillhouse Hollow is an existing dam which has not actually failed. The models used for each case study are indicated in Table 1. The individual documentation for each case study provides detailed descriptive information. A brief description of the dams, reservoirs, and floods associated with each case study is provided below.

Teton Dam Failure Flood

11. The Teton Dam on the Teton River in Idaho failed in June 1976. Eleven lives were lost and damages reportedly were about $400 million. The newly constructed Bureau of Reclamation project was being filled for the first time during the Spring of 1976. The reservoir contained 251,700 acre-feet of water was almost full when the 305-foot-high earthfill dam failed.* Approximately 173,000 acre-feet of water drained through the breached dam within 143 minutes, resulting in a peak discharge of 2.3 million cfs. A total of about 240,000 acre-feet drained from the reservoir within an 8-hour period. In the 100 miles between Teton Dam and the downstream American Falls Reservoir, the peak discharge and flood volume attenuated to 53,500 cfs and 160,000 acre-feet, respectively. American Falls Reservoir stored the entire flood flow. Detailed information complied by the U.S. Geological Survey, including measured discharge data and high water marks, were used in the case study (Ray and Kjelstrom 1978).

12. The Teton Dam breach flood wave is particularly difficult to model due to complex downstream valley geometry. The dam is located in a narrow steep-walled canyon. The canyon ends about 5 miles downstream of the dam

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 4.

site. The river then meanders through a wide, very flat floodplain. Further downstream, the valley abruptly transitions back through narrow constrictions.

13. The Teton Dam case study, as reported in Volume II of this report, consisted of developing original input data from topographic maps and basic reservoir and dam data. A set of Teton data is also routinely provided by the National Weather Service and The Hydrologic Engineering Center as test data with the DAMBRK computer program. Part of Captain Buttery's work involved application of the hydrologic and simplified dynamic routing models using this test data set. The results of this work are summarized in this main report and documented in detail by Buttery (1984).

Hypothetical Prismatic Channel

14. The downstream valley geometry was the dominant consideration in the Teton case study. A hypothetical case study was developed to test the models in an environment in which irregular valley geometry was not the overriding concern. The reservoir data was taken from the Teton case study. However, the downstream valley was replaced with a prismatic channel. The prismatic channel is a hypothetical extension of the 5-mile-long Teton canyon to 50 miles. The prismatic channel section consists of two reaches of constant cross section. The channel from mile 0 to 5 is constant cross section, and the reach between miles 10 and 50 is a slightly wider constant cross section. Miles 5 to 10 provide the transition between the two sections. The cross section for miles 0 to 5 approximates the geometry of the Teton canyon just below the dam. However, the remaining 45 miles of slightly wider prismatic channel is much different from the wide, flat, abruptly changing topography of the Teton Valley below the canyon mouth.

Laurel Run Dam Failure Flood

15. The failure of the dam on Laurel Run near Johnstown, Pennsylvania resulted in the sudden release of 450 acre-feet of water into a stream that was already flooding from a severe rainstorm. Laurel Run has a drainage area of 14 square miles above its confluence with the Conemaugh River. The dam was located 2.5 miles upstream of the confluence. On 19 and 20 July 1977, a severe rainstorm caused heavy flooding in many areas near Johnstown. Flooding in the

Laurel Run Valley caused extensive property damage and loss of more than 40 lives. The 45-foot-high Laurel Run Dam breached about 2:35 a.m. on July 20, significantly worsening flood flows. The data used in the case study analysis came primarily from a paper by Chen and Armbruster (1980) and a report by Land (1980).

Stillhouse Hollow Dam

16. Stillhouse Hollow Dam is located on the Lampases River near Fort Hood in central Texas. The 200-foot-high earthfill embankment impounds a 204,900 acre-foot conservation pool and 390,600 acre-foot flood control pool. Stillhouse Hollow is an existing dam which has not failed. It was selected as a case study because of its well defined, gently changing downstream valley topography and other characteristics which facilitate dam breach flood wave modeling. The project was constructed and is maintained by the Fort Worth District of the Corps of Engineers. The necessary reservoir and dam data were provided by the district office.

Comparison of Case Studies

17. The four case studies represent a broad range of conditions. The Teton case study dealt with extremely difficult valley geometry which included a steep walled canyon, wide flat flood plains, and abruptly changing steep walled constrictions. Laurel Run is in a steep walled valley with fairly irregular geometry in a mountainous area. The Stillhouse Hollow case study dealt with a well-defined smoothly changing valley in an area of rolling hills topography. The hypothetical prismatic channel case study, of course, dealt with a prismatic channel.

18. Laurel Run has a steep channel bottom slope which causes supercritical flow. The other case studies involve relatively mild channel slopes. Laurel Run is also the only case study in which the dam breach occurred during a major rainstorm flood. The four case studies are further compared in Table 2.

Table 2

Comparison of Case Study Dams

Dam	Teton	Laurel Run	Stillhouse Hollow
Type of dam	earth	earth	earth
Dam height (feet)	305	45	200
Storage at time of failure (acre-feet)	252,000	450	1,000,000
Channel slope (feet per mile)	11	98	5.4
Length of valley modeled (miles)	102	2.5	33

PART III: DAM-BREACH FLOOD WAVE MODELS

19. Based on the state-of-the-art review documented by Military Hydrology Report 9 (Wurbs 1985), DAMBRK, FLOW SIM 1 and 2, HEC-1, SMPDBK, HEC Dimensionless Graphs, and TR66 were selected for detailed study. The models are all readily available from Federal agencies and are considered to be representative of the current state-of-the-art of dam breach flood wave modeling. The seven alternative models were developed within the last decade for civilian application. Military Hydrology Bulletins 9 and 10 and the Defense Intelligence Agency (DIA) outflow hydrograph computation method were developed during the 1950's and early 1960's for military application. These methods are now outdated due to recent developments incorporated in more advanced models. However, they were considered to a limited extent in the present study.

20. DAMBRK, FLOW SIM 1, and FLOW SIM 2 are dynamic wave models. DAMBRK and FLOW SIM 2 solve the St. Venant equations using a weighted four-point implicit finite difference scheme. FLOW SIM 1 uses an explicit solution. SMPDBK and the HEC Dimensionless Graphs are simplified methods which make use of generalized relationships previously developed using dynamic wave models. HEC-1 and TR66 use nondynamic wave routing methods.

21. The models are briefly described below. References providing detailed model documentation are cited.

Dam-Break Flood Forecasting Model (DAMBRK)

22. Computer program DAMBRK was developed by Dr. Danny L. Fread of the National Weather Service (NWS) Hydrologic Research Laboratory in Silver Spring, Maryland. The version of DAMBRK used in this study is described in detail by Fread (1983). The Hydrologic Engineering Center (HEC) maintains current versions of the program for use by Corps of Engineers personnel. In addition, the HEC has modified the program's data input structure, developed several utility programs for processing input data and displaying computed results, and prepared a user's manual (HEC 1981). Both the HEC modified and original NWS versions of the model were obtained from the HEC. The NWS version was selected for use in the investigation. This version was a little easier to load into the computer because it does not have the HEC preprocessor program. The DAMBRK version A dated 30 January 1982 used in this study has been

superseded by a version dated 18 July 1984 (Fread 1984). Several significant new options and program changes were made. However, the results obtained and lessons learned from the case studies are still valid regardless of the version of the model used.

23. A user option provides for reservoir routing to be either hydrologic or dynamic. Two types of breaching may be simulated. An overtopping failure is simulated as a rectangular, triangular, or trapezoidal shaped opening that grows progressively downward from the dam crest with time. Flow through the breach at any instant is calculated using a broad-crested weir equation. A piping failure is simulated as a rectangular orifice that grows with time and is centered at any specified elevation within the dam. Weir and orifice flows include corrections that account for tailwater submergence. The pool elevation at which breaching begins, the time required for beach formation, and the geometric parameters of the breach must be specified by the user. The study reported herein used an overtopping failure and hydrologic reservoir routing in all cases.

24. The outflow hydrograph from the reservoir is routed downstream by a weighted four-point implicit nonlinear finite difference solution of the St. Venant equations. Dynamic routing is the only option for the downstream valley. The input data for valley cross-sections can specify inactive as well as active flow areas. The inactive portion of a cross-section is intended to account for an area where water ponds and/or does not have a significant velocity component in the direction of flow.

25. DAMBRK can simulate the progression of a dam-break wave through a downstream valley containing one or more additional dams that may or may not fail. However, the multiple dams have to be in series. The present study was limited to a single dam.

Flow Simulation Models (FLOW SIM 1 and 2)

26. The FLOW SIM 1 and 2 computer programs were developed by B. R. Bodine of the Southwestern Division of the Corps of Engineers in Dallas, Texas. Several district offices of the Southwestern Division have been routinely using the models in their dam safety studies. A user's manual (Bodine, undated) provides instructions for using the programs. The computer programs are generalized models for simulating unsteady and spatially varied flow in rivers

and for simulating dam failures. Both versions provide dynamic routing. The two alternative models use identically the same input data. The difference between the models is that FLOW SIM 1 uses an explicit solution scheme and FLOW SIM 2 uses an implicit solution scheme for solving the St. Venant equations. The computer programs used in the study were obtained directly from the Southwestern Division. Although a few initial runs were made with an earlier version of the models, all of the results reported herein were obtained with the December 1983 edition of both models.

27. A user option provides for reservoir routing to be either hydrologic or dynamic. Two breach routines are provided. A breach routine similar to DAMBRK and HEC-1, in which breach dimensions vary linearly with time, is provided. The other option is an erosion type breaching technique in which the rate of growth of a trapezoidal-shaped breach is estimated using the Schoklitsch erosion formula. The study reported herein used hydrologic reservoir routing with breach dimensions varying linearly with time.

28. Dynamic routing is the only option for downstream valley routing. Inactive as well as active flow areas can be included in the cross-section data.

29. Although not used in this study, FLOW SIM 1 and 2 have the capability for simulation of multiple-tributary, branching stream systems. Multiple dam failures can be simulated with the dams located on different tributaries as well as in series.

Flood Hydrograph Package (HEC-1)

30. The widely used HEC-1 computer program, developed by the Hydrologic Engineering Center, models the precipitation-runoff process and routes flood hydrographs through channels and reservoirs. The package has economic flood damage analysis, flood control system optimization, and dam safety analysis capabilities. The dam safety analysis capability can be used to evaluate the overtopping potential of a dam and to analyze the flood wave that would result from an assumed structural failure of a dam. Program documentation includes a detailed users manual (HEC 1981). The 30 October 1981 version of the program was used in this study.

31. HEC-1 contains a breach simulation methodology, similar to DAMBRK and FLOW SIM 1 and 2, in which breach dimensions grow linearly in time. The

hydrologic reservoir routing is also essentially the same as DAMBRK and FLOW SIM 1 and 2. Several hydrologic channel routing options are provided. Modified Puls routing was used in this study. Two options are provided for handling the storage versus outflow relationships used in modified Puls channel routing. The storage versus outflow relationships can be derived from water surface profile studies or other hydraulic analysis and supplied as input data to HEC-1. Alternatively, a cross-section representative of a routing reach can be furnished as input data. Outflows and storages are computed by the model, assuming the representative cross section is constant through the reach and uniform flow. This latter option was used in this study.

32. The hydrologic routing methods in HEC-1 do not reflect backwater effects. Tailwater conditions are not considered in developing the reservoir outflow hydrograph.

Simplified Dam-Break Flood Forecasting Model (SMPDBK)

33. Jonathan N. Wetmore and Danny L. Fread of the Hydrologic Research Laboratory, NWS, developed the SMPDBK model for use in dam-failure analyses when time is limited or where main frame computer facilities are unavailable to the user. The model was first presented by Wetmore and Fread (1981). Wetmore and Fread (1983) present a brief outline of the model's conceptual basis, a step-by-step guide and example of the computations involved in the model, and listings of FORTRAN and BASIC computer codes.

34. The objective in developing SMPDBK was to retain the critical aspects of the DAMBRK model while eliminating the need for large computer facilities. This is accomplished by assuming the downstream channel to be prismatic; neglecting the effects of off-channel storage; determining only the peak flows, stages, and travel times; neglecting the effects of backwater from downstream bridges and dams; and utilizing dimensionless peak-flow routing graphs developed using DAMBRK. The SMPDBK procedure consists of three steps: (a) approximation of the channel downstream of the dam as a prismatic channel, (b) calculation of the peak outflow at the dam using the temporal and geometrical description of the breach and the reservoir volume, and (c) calculation of dimensionless routing parameters used with dimensionless routing curves to determine the peak flow at specified cross sections downstream of the dam.

35. The computations can be done manually using graphs developed by Wetmore and Fread, and several computer versions are available. An Apple microcomputer version was used in this study. This version dated May 1983 was adapted to the Apple microcomputer by Mr. David Brandon of the National Weather Service Forecast Office in Topeka, Kansas.

Dimensionless Graphs

36. Dimensionless graphs were developed by Dr. John G. Sakkas, Consulting Engineer, Davis, California, for the Hydrologic Engineering Center. The graphs are documented by a report prepared by Dr. Sakkas (1974), which was subsequently published as HEC Research Note No. 8 (Sakkas 1980).

37. A computation method based on the method of characteristics was used to solve the St. Venant equations. The graphs were developed for a dry prismatic channel assuming instantaneous complete removal of the dam. Graphs were prepared for several of the parameters involved to cover a practical range of conditions.

38. The graphs can be used to estimate time of wave front arrival, maximum flood depth, and time of maximum flood depth at selected distances downstream of the dam. A procedure is provided for transforming irregular natural cross sections into one representative prismatic section of either rectangular, triangular, or parabolic cross-sectional shape. Characteristics of the channel section and the water depth behind the dam are used to compute the parameters required to use the graphs.

Simplified Dam-Breach Routing Procedure (TR 66)

39. Soil Conservation Service (SCS) Technical Release (TR) No. 66 dated December 1981 (revised version) was developed by John A. Brevard and Fred D. Theurer of the Engineering Division in Glenn Dale, Maryland. The TR 66 simplified routing procedure is based upon the attenuation-kinematic routing model which is described by Comer, Theurer, and Richardson (1982).

40. TR 66 is a simplified step-by-step manual computation procedure, based on graphs, for routing a dam-breach flood wave. Input data required to use the procedure consists of valley cross sections with depth versus discharge and depth versus area curves at each section, reservoir storage volume, and

depth of water behind the dam. The procedure is used to compute peak discharges and associated stages at each valley section.

41. An instantaneous breach is assumed. A standard triangular or curvilinear breach hydrograph shape is assumed, depending on whether the flow in the channel reach immediately below the dam is supercritical or subcritical, respectively. The maximum discharge at the dam is determined from a curve of maximum discharge versus reservoir depth developed on the basis of information from actual dam failures. The downstream routing is a simplified version of the attenuation-kinematic model, which is a simultaneous storage routing-kinematic routing method. The model reflects attenuation due to valley storage characteristics and the timing and distortion of the flood wave due to kinematic translation.

Military Hydrology Bulletins 9 and 10

42. The Military Hydrology Research and Development Branch of the Washington District of the US Army Corps of Engineers investigated dam-breach flood forecasting methods during the 1950's. Several reports were prepared, including Military Hydrology Bulletins 9 and 10 (USACE 1957). Bulletin 9 outlines step-by-step graphical procedures for determining the outflow hydrograph from a breached dam using empirical weir and orifice formulas and hydrologic storage routing. Bulletin 10 presents similar procedures for determining the reservoir outflow hydrograph but also includes step-by-step procedures for downstream routing using the Muskingum method. These simplified methods were developed to permit rapid flood wave analysis with a degree of accuracy acceptable for military applications.

DIA Outflow Hydrograph Computation Method

43. A report published by the Defense Intelligence Agency (1963) updated Military Hydrology Bulletins 9 and 10 to better account for a negative wave in the reservoir and tailwater. The manual step-by-step procedure is designed for use by military engineers in expeditiously computing the outflow hydrograph from a breached dam. An instantaneous rectangular breach is assumed. Downstream routing is not included in the method.

PART IV: SUMMARY OF CASE STUDY RESULTS

44. The scope and results of the case study analyses are described here from an overview perspective. The intent in this main report is to focus on general observations and conclusions regarding the lessons learned from the case studies. Detailed documentation of each case study is provided by the other volumes of this report and the report by Buttery (1984).

45. The four case studies and nine models are listed in Table 1. This table shows which of the models were applied in each case study. An original best estimate set of input data was developed for each case study. A "base run" with each model was made with input data as close as possible to the best estimate data set. The results from the base runs were used for comparison with measured data and between models. Numerous other runs were made for purposes of sensitivity analysis and otherwise testing the models. The models provide various types of information regarding the flood wave characteristics. The results were summarized in terms of peak discharges, peak water surface elevations, maximum flow depths, and time to maximum depth at various distances from the dam.

Computer Resources

46. The four models requiring a mainframe computer were run on the Amdahl 470 computer system at Texas A&M University. Memory requirements for DAMBRK, FLOW SIM 1, FLOW SIM 2, and HEC-1 were 436K, 452K, 472K, and 556K bytes, respectively. As an indication of the relative magnitude of central processor unit (CPU) resources required by these programs, the Laurel Run base runs with DAMBRK, FLOW SIM 1, FLOW SIM 2, and HEC-1 had CPU times of 103 seconds, 248 seconds, 131 seconds, and 10 seconds, respectively. The Teton base runs with DAMBRK, FLOW SIM 1, and HEC-1 had CPU times of 166 seconds, 106 seconds, and 18 seconds, respectively. Throughout the study, computer runs were made on low priority which meant a long turnaround time, usually overnight, but very modest costs. Successful runs of DAMBRK, FLOW SIM 1, and FLOW SIM 2 had computer charges of $1 to $3. Unsuccessful runs were $0.20 to $1.00. HEC-1 ran for less than a dollar with most of the cost for printing.

47. Although the number of runs made with the models was not precisely counted, DAMBRK, FLOW SIM 1, and FLOW SIM 2 ran a total of well in excess of

500 times. Most of these runs did not result in a solution due to nonlinear instabilities or other numerical computation problems or in some cases, user error. Relatively few runs were made with HEC-1 because the program almost always resulted in a solution the first try.

48. SMPDBK was run on an Apple microcomputer, with relatively short run times. The other models involved manual computations with graphs and a calculator.

Numerical Computation Difficulties

49. Most of the runs made with the three dynamic routing models (DAMBRK, FLOW SIM 1, and FLOW SIM 2) terminated without reaching a solution due to nonconvergence or instability in the calculations. Overcoming these dynamic routing computational problems was the most difficult and time-consuming aspect of the case studies. The modeling process consists of first developing a best estimate set of input data. The input data is then modified in a trial and error manner with alternative combinations of parameter values being tried until a solution is obtained. The objective is to obtain a solution with the input data being as close as possible to the best estimate data.

50. The literature in general, as well as the experience with the case studies reported herein, indicates that rapidly rising hydrographs such as the dam-breach outflow hydrograph can be expected to cause computational problems, associated with instabilities and the iterative solution algorithm terminating without converging to a solution, when modeled using numerical approximations of the St. Venant equations. These problems are usually associated with the distance and time steps used in the computations and/or abruptly changing valley geometry. DAMBRK prints out a message that "nonconvergence occurred at certain cross-sections" whenever the iterative Newton-Raphson technique does not converge to a solution. FLOW SIM 1 and 2 print out a message that "execution of the program is terminated because of an instability in the calculations."

51. Since flow characteristics are dependent upon downstream conditions in subcritical flow but not in supercritical flow, occurrence of supercritical flow is also a problem. DAMBRK can accommodate supercritical for either the entire channel or only an upstream reach of the channel, but the flow is assumed to be supercritical throughout the duration of the simulation.

Changes between subcritical and supercritical flow as the flood wave passes a location cannot be modeled. FLOW SIM 1 and 2 have no special provisions for supercritical flow.

52. Froude numbers too much above unity combined with valley geometry changing too abruptly can cause the computations to terminate without a reasonable solution. The two types of computational problems are closely related.

53. Varying the distance and time steps and smoothing the valley geometry are the primary means of overcoming instability and nonconvergence problems. The flow in the channel at the beginning of the simulation can also be arbitrarily increased to prevent negative flow depths from occurring. Smoothing the valley geometry consists of removing abrupt changes either vertically or along the channel by altering topwidth-elevation data or relocating or removing cross sections. Increasing the value of the Manning roughness coefficients is one way of preventing supercritical flow from occurring. Convergence to a reasonable solution was also found in the case studies to be very sensitive to breach formation time and final breach width. If the water is allowed to flow from the reservoir relatively slowly (small breach width and large breach time) the likelihood of obtaining a solution is better than for a large, rapidly formed breach.

54. The valley downstream of Teton Dam required significant cross-sectional smoothing to obtain a solution. The initial attempt at modeling the Teton flood wave using DAMBRK was unsuccessful in that convergence to a reasonable solution was never obtained. Numerous runs with trial-and-error adjustments in input data terminated with messages that nonconvergence had occurred or negative areas had been computed. In some cases, solutions were obtained but were unreasonable. The adjustments including smoothing, relocating, or removing selected cross-sections; increasing the base discharge; changing the weighting factor; and relaxing the convergence criterion. Finally, the initial data set was abandoned and an essentially new data set developed. The second data set had fewer cross sections, fewer topwidths per cross section, and different cross-section locations. Each and every cross section was smoothed. Initial runs with the second data set did not converge, but minor additional smoothing resulted in convergence to a reasonable solution. Although a solution was obtained with the base run (best estimate) input data, solutions still could not be obtained for very large breach widths, very small breach times, or smaller Manning roughness coefficient values.

55. In regard to overcoming computational problems to obtain a reasonable solution, the performance of FLOW SIM 1 in the Teton case study was essentially the same as DAMBRK. A base run solution was obtained using the same smoothed valley geometry as used with DAMBRK. However, alternative runs with variations in breach parameters and other input data would terminate with a message indicating computational instability had occurred. The input data which ran successfully in FLOW SIM 1 would not run in FLOW SIM 2. Numerous unsuccessful runs were made with various combinations of input data. The best solution actually obtained included a breach time of five hours which was considered to not be reasonably close to the one-hour base run breach time.

56. The hypothetical prismatic channel case study eliminated the abruptly changing valley geometry. Obtaining a solution with DAMBRK was no problem under these conditions. However, computational instability was still a major problem with FLOW SIM 1 and 2. Trial and error runs were made with various combinations of values for the time and distance step sizes, breach characteristics, and Manning roughness coefficients. The distance and time step sizes did not seem to make much difference. Reasonable solutions could be obtained with FLOW SIM 1 as long as either the breach width was relatively small, breach time was relatively large, and/or the Manning roughness coefficients were relatively large. Input data which resulted in solutions with FLOW SIM 1 terminated due to computational instability with FLOW SIM 2. The input parameters mentioned above had to have extremely favorable values to obtain a solution with FLOW SIM 2. Serious computational difficulties were not anticipated to occur with the prismatic channel case study. The investigator never developed a satisfactory understanding of why the problems with FLOW SIM 1 and 2 were occurring. It could have been due to user error or lack of skill in applying the programs or weaknesses in the programs. The transition from a relatively flat canyon floor to steep canyon walls may have caused the computational problems. The right combination of distance and time step sizes may have never been obtained.

57. Although significant smoothing was still required, the Laurel Run valley geometry was much easier to model than the valley below the Teton Dam in regard to cross-section data. However, Laurel Run has the additional complication of a steep channel bottom slope of roughly 100 feet per mile. In order to obtain a solution with the dynamic routing models, the Manning roughness coefficients were increased enough to prevent supercritical flow from

occurring. Significant time and effort involving numerous computer runs were required to overcome nonconvergence and instability problems. Cross-section data, distance and time steps, roughness coefficients, and other input data were varied in a trial-and-error manner in an attempt to obtain a solution with input data as close to the original best estimate data as possible. The best runs obtained with DAMBRK and FLOW SIM 1 included Manning roughness coefficients which were double the actual values. A solution was obtained with FLOW SIM 2 with roughness coefficients which were 1.5 times the actual estimated values. FLOW SIM 2 performed a little better than FLOW SIM 1 and DAMBRK in obtaining solutions in the Laurel Run case study.

58. In regard to the Stillhouse Hollow case study, a solution was readily obtained with DAMBRK. Although several runs were required to properly adjust values of the distance step sizes and other input data, nonconvergence and instability were not a major problem. Buttery (1984) developed input data and made several runs with FLOW SIM 1 without obtaining a solution. The author later attempted to obtain solutions with FLOW SIM 1 and 2 but was unsuccessful. The program continued to terminate due to instability in the calculations for various combinations of input data. However, the FLOW SIM 1 and 2 analysis for the Stillhouse Hollow case study was not pursued nearly as extensively as for the other case studies.

Comparative Summary of Model Results

59. The results obtained by applying the various models to the case study data sets are summarized in Tables 3 through 17 in terms of peak discharge, peak flow depth, and time to peak flow depth at selected locations along the streams. Peak discharges are plotted in Figures 1 through 5. The Teton Dam and Laurel Run Dam actually failed, and field measurements of the resulting flood wave characteristics are available. Although the field measurements are somewhat imprecise, an opportunity is provided to test the accuracy of the models. The tables compare computed to measured peak discharges and times to peak stage by expressing computed values as a percentage of measured values (computed value divided by measured value times 100%). Peak flow depths are expressed in terms of deviation from high-water marks in feet (computed depth minus measured depth). Although measured flood data do not exist for the Stillhouse Hollow and hypothetical prismatic channel case studies,

Table 3
Peak Discharges for Teton

Model	Distance Below Dam in Miles: 2.5	8.8	55.7	67.5
	Peak Discharge in 1000 cfs			
Measured	2,300	1,060	90.5	67.3
DAMBRK	1,890	1,020	203	200
FLOW SIM 1	1,670	950	150	150
HEC-1	1,760	1,200	245	220
SMPDBK	2,220	1,740	420	420
	Percent of Measured Peak Discharge			
Measured	100%	100%	100%	100%
DAMBRK	82%	96%	224%	297%
FLOW SIM 1	73%	90%	167%	223%
HEC-1	76%	113%	270%	327%
SMPDBK	97%	164%	464%	620%

Table 4
Peak Flow Depths for Teton

Model	Distance Below Dam in Miles: 2.5	8.9	20.0	35.5	53.8	67.5	90.0
	Peak Flow Depth in Feet						
Measured	---	18	27	13	20	12	13
DAMBRK	64	22	23	17	28	29	17
FLOW SIM 1	63	21	22	16	25	27	11
HEC-1	78	40	37	29	38	72	30
SMPDBK	75	54	23	29	38	35	---
	Deviation from High Water Marks in Feet						
Measured	---	0	0	0	0	0	0
DAMBRK	---	4	-4	4	8	17	4
FLOW SIM 1	---	3	-5	3	5	15	-2
HEC-1	---	22	10	16	18	60	17
SMPDBK	---	36	-4	16	18	23	---

Table 5
Time to Peak Flow Depth for Teton

Model	Distance Below Dam in Miles: 2.5	8.9	20.0	35.0	53.8	67.5	90.0
	Time to Peak Flow Depth in Hours						
Measured	2.0	2.5	---	---	31	36	58
DAMBRK	1.1	2.7	6.0	12.6	25.0	27.7	36.0
FLOW SIM 1	4.0	5.9	9.2	15.4	29.1	31.6	41.2
HEC-1	1.2	1.8	4.5	8.5	12.9	16.0	21.0
SMPDBK	1.2	2.0	4.5	9.2	16.1	18.3	---
	Percent of Measured Time						
Measured	100%	100%	---	---	100%	100%	100%
DAMBRK	55%	108%	---	---	81%	77%	62%
FLOW SIM 1	200%	236%	---	---	94%	88%	71%
HEC-1	60%	72%	---	---	42%	44%	36%
SMPDBK	60%	80%	---	---	52%	51%	---

Table 6
Peak Discharges
DAMBRK Test Data for Teton

Model	Distance Below Dam in Miles: 0.0	5.0	8.5	32.5	37.5	43.0	51.5	59.5
	Peak Discharge in 1000 cfs							
DAMBRK	1,644	969	894	178	122	100	81	66
HEC-1	1,227	1,166	865	284	277	257	251	--
SMPDBK	1,632	1,318	1,022	226	196	189	200	205
TR66	1,929	1,321	1,051	270	222	186	166	154
Bul 9&10	1,648	1,135	1,046	786	778	879	443	396
	Percent of DAMBRK Peak Discharge							
DAMBRK	100%	100%	100%	100%	100%	100%	100%	100%
HEC-1	75%	120%	97%	160%	227%	257%	310%	----
SMPDBK	99%	136%	114%	127%	161%	189%	247%	311%
TR66	117%	136%	118%	152%	182%	186%	205%	233%
Bul 9&10	100%	117%	117%	442%	638%	879%	547%	600%

Table 7
Peak Flow Depths
DAMBRK Test Data for Teton

Model	Distance Below Dam in Miles: 0.0	5.0	8.5	32.5	37.5	43.0	51.5	59.5
			Peak Flow Depth in Feet					
DAMBRK	96.2	58.7	29.8	21.2	21.8	27.2	27.6	17.8
HEC-1	91.4	71.9	26.7	21.1	21.5	33.6	52.6	----
SMPDBK	103.2	66.0	31.4	21.9	26.2	34.5	42.4	33.0
TR66	127.0	87.5	36.2	21.2	26.2	36.2	41.0	30.5
Bul 9&10	103.3	73.1	35.8	26.1	31.5	67.6	53.3	38.5
			Deviation from DAMBRK Peak Depth in Feet					
DAMBRK	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
HEC-1	-4.8	13.2	-3.1	-0.1	-0.3	6.4	25.0	---
SMPDBK	7.0	7.3	1.6	0.7	4.4	7.3	14.8	15.2
TR66	30.8	28.8	6.4	0.0	4.4	9.0	13.4	12.7
Bul 9&10	7.1	14.4	6.0	4.9	9.7	40.4	25.7	37.5

Table 8
Time to Peak Flow Depth
DAMBRK Test Data for Teton

Model	Distance Below Dam in Miles: 0.0	5.0	8.5	32.5	37.5	43.0	51.5	59.5
			Time to Peak Flow Depth in Hours					
DAMBRK	1.33	2.63	3.44	20.85	27.53	31.37	33.21	33.97
HEC-1	1.75	2.00	2.75	9.25	10.00	11.75	12.50	----
SMPDBK	1.3	1.7	2.4	8.4	10.1	11.4	12.7	14.2
Bul 9&10	0.00	1.33	1.81	11.15	13.68	16.68	18.97	23.6
			Percent of DAMBRK Peak Discharge					
DAMBRK	100%	100%	100%	100%	100%	100%	100%	100%
HEC-1	132%	76%	80%	44%	36%	37%	38%	----
SMPDBK	98%	65%	70%	40%	37%	36%	38%	42%
Bul 9&10	0%	51%	53%	53%	50%	53%	57%	69%

Table 9
Peak Discharges for Prismatic Channel

Model	0	5	10	20	30	40	50
	Distance Below Dam in Miles						
	Peak Discharge in 1000 cfs						
DAMBRK	3,841	3,468	3,220	2,529	2,135	1,777	1,567
HEC-1	3,911	3,558	3,291	2,910	2,520	2,180	1,856
SMPDBK	4,016	3,235	3,212	3,880	2,620	2,410	2,216
TR66 1/	3,841	2,996	2,612	1,950	1,540	1,300	1,044
	Percent of DAMBRK Peak Discharge						
DAMBRK	100%	100%	100%	100%	100%	100%	100%
HEC-1	102%	103%	102%	115%	118%	123%	118%
SMPDBK	105%	93%	100%	114%	123%	136%	141%
TR66 1/	100%	86%	81%	77%	72%	73%	67%

1/ The TR66 procedure resulted in a peak discharge at the dam of 1,931,000 cfs which is much smaller than the values obtained with the other models. Consequently, the DAMBRK peak discharge at the dam of 3,841,000 cfs was used with all other computations performed with the TR66 procedure.

Table 10
Peak Flow Depths for Prismatic Channel

Model	0	5	10	20	30	40	50
	Distance Below Dam in Miles						
	Peak Flow Depth in Feet						
DAMBRK	114.6	103.9	93.6	83.9	75.3	68.9	53.0
HEC-1	-----	-----	145.1	135.8	136.0	137.8	147.5
SMPDBK	112.6	100.2	89.5	84.0	79.9	76.7	73.0
TR66	123.0	107.9	89.0	75.3	66.4	61.0	54.4
Graphs	119.5	101.6	95.8	87.7	81.8	78.1	72.8
	Deviation from DAMBRK Peak Flow Depth in Feet						
DAMBRK	0.0	0.0	0.0	0.0	0.0	0.0	0.0
HEC-1	---	--	51.5	51.9	60.7	68.9	94.5
SMPDBK	-2.0	-3.7	-4.1	0.1	4.6	7.8	20.0
TR66	8.4	4.0	-4.6	-8.6	-8.9	-7.9	1.4
Graphs	4.9	-2.3	2.2	3.8	6.5	9.2	19.8

Table 11
Time to Peak Flow Depth
Prismatic Channel

Model	Distance Below Dam in Miles 0	5	10	20	30	40	50
			Time to Peak Depth in Hours				
DAMBRK	1.00	1.15	1.30	1.65	2.10	2.61	2.99
HEC-1	--	--	1.30	1.72	2.17	2.76	3.13
SMPDBK	1.00	1.20	1.50	1.97	2.45	2.98	3.50
Graphs	0.00	0.43	0.68	1.07	1.36	1.75	2.22
			Percent of DAMBRK Time to Peak Depth				
DAMBRK	100%	100%	100%	100%	100%	100%	100%
HEC-1	--	--	100%	104%	103%	105%	105%
SMPDBK	100%	104%	115%	119%	117%	114%	117%
Graphs	0%	37%	52%	65%	65%	67%	74%

Table 12
Peak Discharges for Laurel Run

Model	Peak Discharge One Mile Below Dam	Percent of Measured
Measured	37,000 cfs	--
DAMBRK	36,500 cfs	99%
FLOW SIM 1	34,800 cfs	94%
FLOW SIM 2	32,900 cfs	89%
HEC-1	37,800 cfs	102%
SMPDBK	53,800 cfs	145%

Table 13
Peak Flow Depths for Laurel Run

Model	Distance Below Dam in Feet						
	400	2,000	4,000	6,000	8,000	10,000	12,000
			Peak Flow Depth in Feet				
Measured	11.0	16.0	17.5	18.5	15.0	14.0	17.0
DAMBRK	16.2	17.1	17.7	15.7	21.1	18.2	16.4
FLOW SIM 1	14.7	17.9	20.0	15.3	23.9	20.4	15.5
FLOW SIM 2	11.8	15.5	16.8	13.5	20.8	17.4	13.3
HEC-1	16.1	31.7	24.6	28.9	23.7	22.7	29.2
SMPDBK	10.7	17.4	17.8	16.6	16.3	15.3	25.4
Graphs	23.4	20.5	19.0	18.0	16.4	15.4	15.1
			Deviation from High Water Marks in Feet				
Measured	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DAMBRK	5.2	1.1	0.2	-2.8	6.1	4.2	-0.6
FLOW SIM 1	3.7	1.9	2.5	-3.2	8.9	6.4	-1.5
FLOW SIM 2	0.8	-0.5	-0.7	-5.0	5.8	3.4	-3.7
HEC-1	5.1	15.7	7.1	10.4	8.7	8.7	12.2
SMPDBK	-0.3	1.4	0.3	-1.9	1.3	1.3	8.4
Graphs	12.4	4.5	1.5	-0.5	1.4	1.4	-1.9

Table 14
Time to Peak Flow Depth for Laurel Run

Model	Distance Below Dam in Miles						
	400	2,000	4,000	6,000	8,000	10,000	12,000
			Time to Peak Flow Depth in Hours				
DAMBRK	0.26	0.28	0.33	0.35	0.41	0.44	0.49
FLOW SIM 1	0.35	0.37	0.41	0.44	0.50	0.53	0.60
FLOW SIM 2	0.36	0.38	0.41	0.43	0.48	0.50	0.55
HEC-1	0.25	0.27	0.28	0.32	0.33	0.35	0.38
SMPDBK	0.3	0.3	0.3	0.3	0.4	0.4	0.4
Graphs	0.01	0.03	0.06	0.09	0.12	0.15	0.18

Table 15
Peak Discharges for Stillhouse Hollow

Model	0.0	3.03	4.98	7.48	15.38	21.09	29.02	33.43
				Distance Below Dam in Miles				
				Peak Discharge in 1000 cfs				
DAMBRK	2,783	2,683	2,550	2,477	2,342	2,296	2,220	2,228
HEC-1	2,644	2,511	2,428	2,363	2,258	1,878	1,756	---
SMPDBK	3,827	2,826	2,872	2,870	2,888	2,554	2,203	2,038
TR66	2,783	2,421	2,241	2,060	1,698	1,503	1,239	1,113
Bul 9&10	3,000	2,505	2,376	2,240	1,854	1,695	1,521	1,413
				Percent of DAMBRK Peak Discharge				
DAMBRK	100%	100%	100%	100%	100%	100%	100%	100%
HEC-1	95%	94%	95%	95%	96%	82%	79%	---
SMPDBK	138%	105%	113%	116%	123%	111%	99%	91%
TR66	100%	90%	88%	83%	73%	65%	56%	50%
Bul 9&10	108%	93%	93%	90%	79%	74%	69%	63%

Table 16
Peak Flow Depths for Stillhouse Hollow

Model	0.0	3.03	4.98	7.48	15.38	21.09	29.02	33.43
				Distance Below Dam in Miles				
				Peak Flow Depth in Feet				
DAMBRK	137.8	88.6	91.4	85.5	74.1	67.5	59.5	58.2
HEC-1	77.0	71.8	79.6	82.5	67.4	71.2	56.2	---
SMPDBK	131.2	103.1	93.4	99.0	78.0	82.1	73.6	79.2
TR66	101.0	97.5	76.5	93.0	58.0	62.0	48.5	51.0
Graphs	142.3	132.5	129.2	125.2	116.7	112.1	107.4	105.1
Bul 9&10	99.0	94.4	75.0	92.9	58.8	63.8	50.2	53.9
				Deviation from DAMBRK Peak Depth in Feet				
DAMBRK	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
HEC-1	-60.8	-16.8	-11.8	-3.0	-6.7	3.7	-3.3	---
SMPDBK	-6.6	14.5	2.0	13.5	3.9	14.6	14.1	21.0
TR66	-36.8	8.9	-14.9	7.5	-16.1	-5.5	-11.0	-7.2
Graphs	4.5	43.9	37.8	39.7	42.6	44.6	47.9	46.9
Bul 9&10	-38.8	5.8	-16.4	7.4	-15.3	-3.7	-9.3	-4.3

Table 17
Time to Peak Flow Depth for Stillhouse Hollow

Model	Distance Below Dam in Miles: 0.0	3.03	4.98	7.48	15.38	21.09	29.02	33.43
			Time to Peak Flow Depth in Hours					
DAMBRK	4.2	4.6	4.8	5.0	6.2	6.6	7.6	7.2
HEC-1	4.2	4.6	5.0	5.6	6.4	8.6	9.6	---
SMPDBK	4.0	4.3	4.6	5.0	6.0	6.9	8.5	9.4
Graphs	0.0	0.6	0.9	1.0	1.8	2.4	3.2	3.7
Bul 9&10	0.0	1.7	2.2	3.0	5.6	7.0	8.9	10.0
		Percent of DAMBRK Time to Peak Flow Depth						
DAMBRK	100%	100%	100%	100%	100%	100%	100%	100%
HEC-1	100%	100%	104%	112%	103%	130%	126%	---
SMPDBK	95%	93%	96%	100%	97%	105%	112%	131%
Graphs	0%	13%	19%	20%	29%	36%	42%	51%
Bul 9&10	0%	37%	46%	60%	90%	106%	117%	139%

Table 18
Average Deviation in Peak Flow Depth

Model	Teton	Laurel Run	Teton Test Data	Prismatic Channel	Stillhouse Hollow
DAMBRK	6.8	2.9	0.0	0.0	0.0
FLOW SIM 1	5.5	4.0	---	no	no
FLOW SIM 2	no	2.8	---	no	no
HEC-1	23.8	9.7	7.6	65.5	15.2
SMPDBK	19.4	2.1	7.3	6.0	11.3
TR66	---	---	13.2	6.3	13.5
Graphs	---	3.4	---	7.0	38.5
Bul 9&10	---	---	18.2	---	12.6

Notes:

1. The numbers in the table above are average deviations in feet computed by averaging the absolute values of the deviations in Tables 4, 7, 10, 13, and 16.
2. For the Teton and Laurel Run case studies, the deviation is the maximum water surface elevation computed with the model minus the measured high water elevation.
3. For the Teton test data, prismatic channel, and Stillhouse Hollow Dam case studies, the deviation is the maximum water surface elevation computed using a given model minus that computed using DAMBRK.

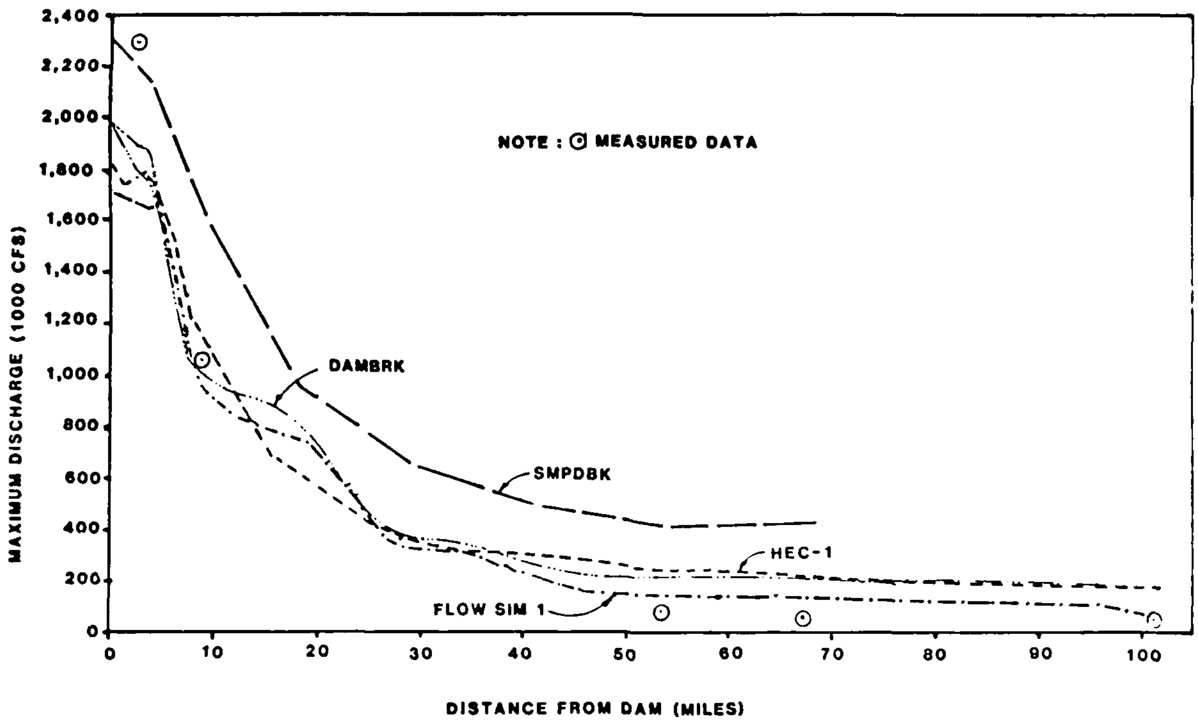

Figure 1. Peak discharges for Teton

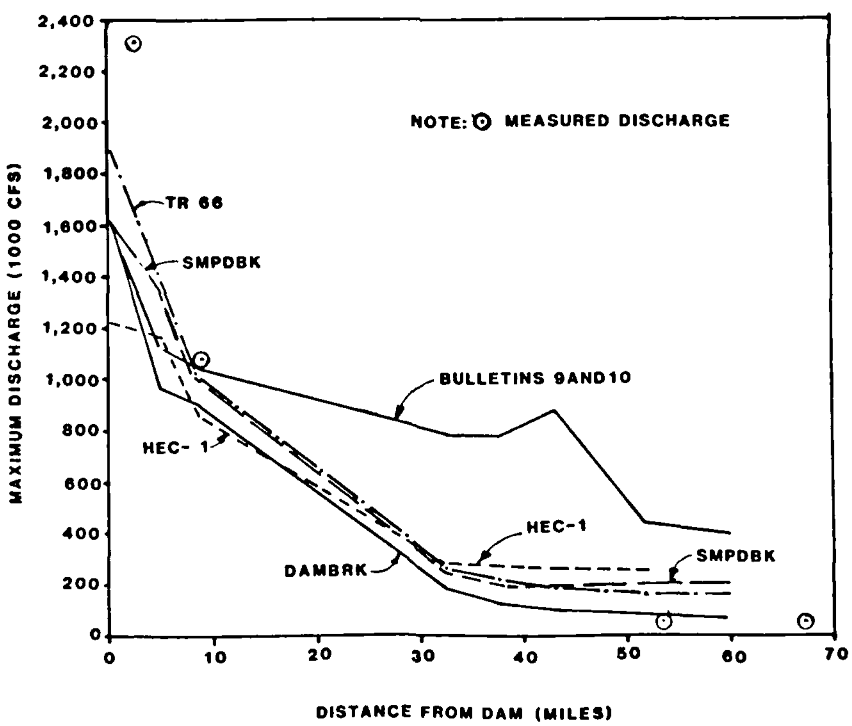

Figure 2. Peak discharges for DAMBRK test data for Teton

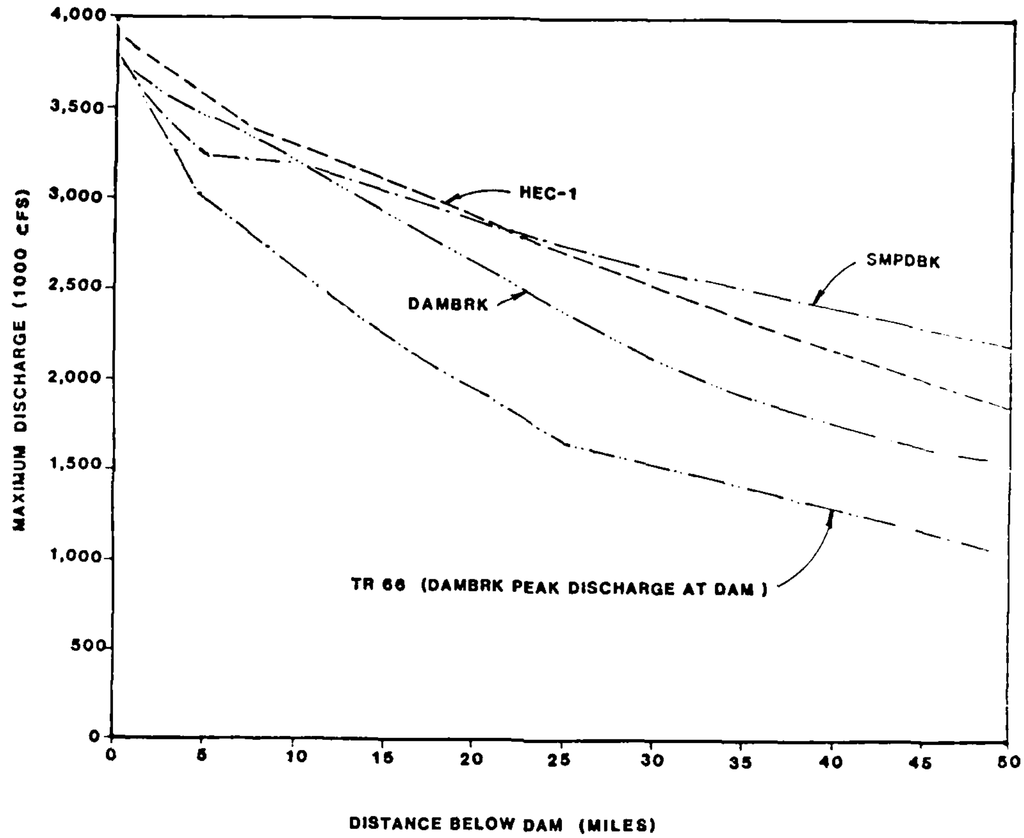

Figure 3. Peak discharges for prismatic channel

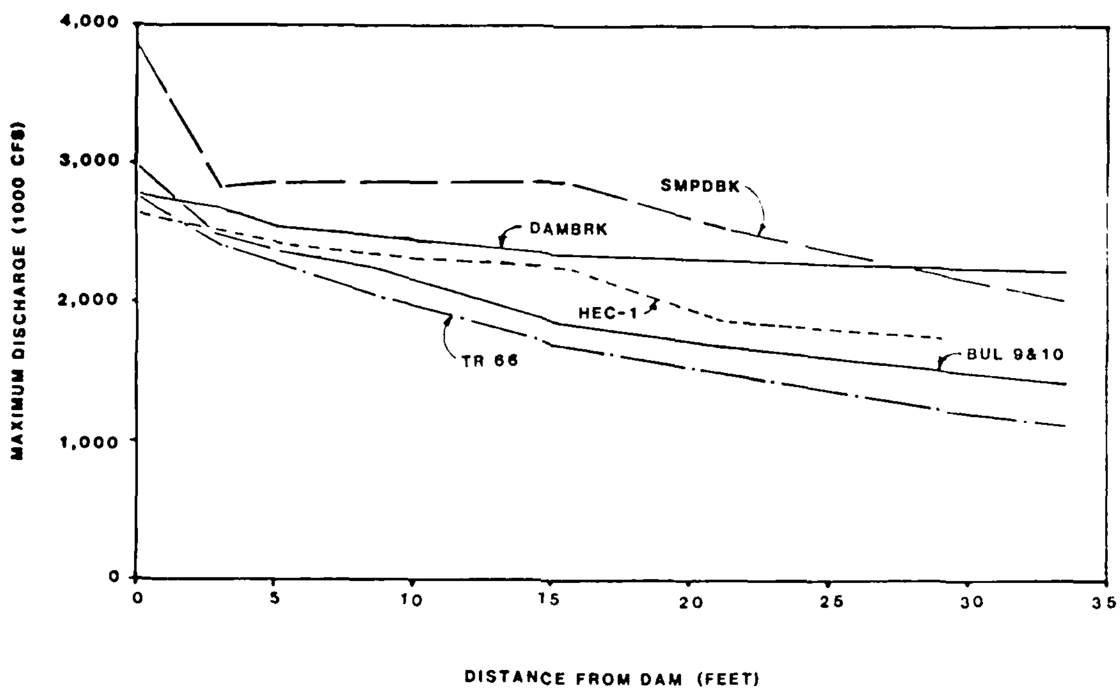

Figure 4. Peak discharges for Stillhouse Hollow

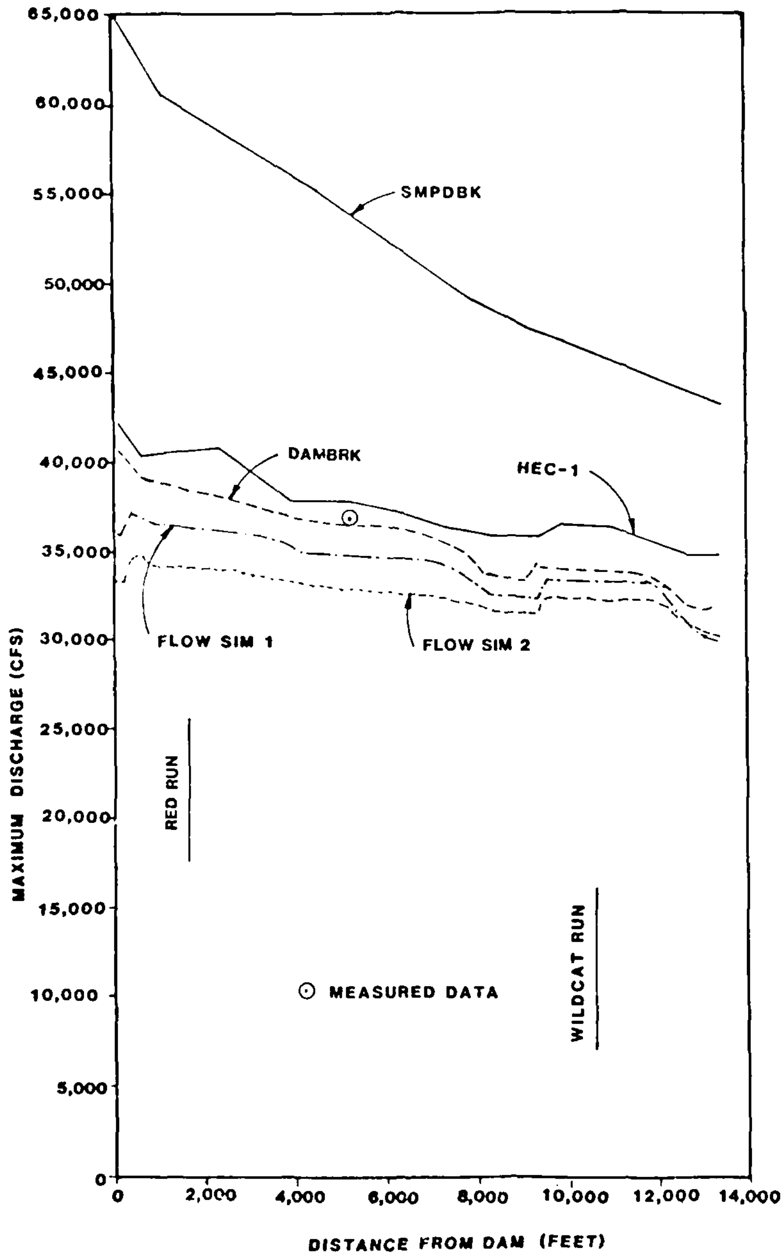

Figure 5. Peak discharges for Laurel Run

comparison of results obtained with the different models still provides a meaningful analysis of model performance. For these case studies, the DAMBRK results are used as a base of comparison. Results from the other models are expressed in the tables as a percentage of or deviation from the DAMBRK results.

60. Indirect measurements of peak discharges for the Teton flood at five locations and the one peak discharge indirect measurement for the Laurel Run flood are indicated in the tables and figures. Field measurements of high-water elevations were available at a number of locations for both the Teton and Laurel Run floods. The field measurements were interpolated as necessary to obtain the high-water elevations at the locations indicated in the tables. The various models have different schemes for determining the channel locations at which results are printed. To facilitate comparison, model results were interpolated as necessary to obtain values for all the models at the same channel locations.

61. Table 18 provides a quantitative measure of the performance of the models in each case study. Average deviation in peak flow depth is used to compare the models. The average deviations were computed by averaging the absolute values of the deviations shown in Tables 4, 7, 10, 13 and 16.

Teton Case Study

62. The Teton Dam case study actually consisted of two data sets. The DAMBRK computer program package obtained from the Hydrologic Engineering Center included a set of test data for use in loading and checking the computer program. These test data are from the Teton flood but may not be completely representative of the flood in all respects. The cross sections are smoothed. The test data were run in the process of loading DAMBRK in the Amdahl and provided a conveniently available data set for other purposes as well. HEC-1, SMPDBK, TR66, and Military Hydrology Bulletins 9 and 10 were applied to the test data set as part of this study. However, an original data set was developed directly from topographic maps and data from the U.S. Geological Survey report completely independently of the test data. DAMBRK, FLOW SIM 1 and 2, HEC-1, and SMPDBK were applied to the original Teton data set.

63. In regard to the original data set, the valley geometry of the Teton and Snake Rivers below the Teton Canyon was significantly smoothed to

obtain convergence to a solution with the dynamic wave models. The results shown were obtained assuming no volume losses. Otherwise, the input data was reasonably close to the best estimate of actual conditions.

64. Table 18 shows that, for the seven locations included in Table 4, the difference between computed and measured flood depths average 5.5 feet, 6.8 feet, 19.4 feet, and 23.8 feet for FLOW SIM 1, DAMBRK, SMPDBK, and HEC-1, respectively. Thus, the differences between computed and measured peak water surface profiles are large for all four models. FLOW SIM 1 and DAMBRK were significantly more accurate than SMPDBK and HEC-1. Peak discharges computed with DAMBRK were 82 to 297 percent of the measured peak discharges. FLOW SIM 1, HEC-1, and SMPDBK had peak discharges of 73 to 223 percent, 76 to 327 percent, and 97 to 620 percent, respectively, of the measured values. Table 5 shows that the computed times to peak depth are significantly less than measured values. The FLOW SIM 1 times to peak shown in the table are actually not comparable to the other data because the breach began to form about two hours after time zero when the simulation began.

65. FLOW SIM 2 is not shown because a solution was not obtained due to computational instability. The dimensionless graphs were considered to be almost meaningless for the extremely nonprismatic valley of the Teton and Snake Rivers.

66. For the Teton data furnished as test data with DAMBRK, the average deviations in peak flow depth shown in Table 18 are based on a comparison with the DAMBRK values. SMPDBK, HEC-1, TR66, and Military Hydrology Bulletins 9 and 10 resulted in peak flows depths averaging 7.3 feet, 7.6 feet, 13.2 feet, and 18.2 feet, respectively, higher than those computed with DAMBRK. Peak discharge and time to peak depth variations are shown in Tables 6 and 8.

Hypothetical Prismatic Channel Case Study

67. Even with a prismatic channel, the results obtained with the alternative models varied significantly. The peak flow depths computed using the alternative models are compared in Tables 10 and 18 using the DAMBRK results as a base for comparison. The DAMBRK peak depths for the seven locations shown in the table average 84.7 feet. Deviations from the DAMBRK peak depths average [illegible].0 feet, 6.3 feet, 7.0 feet, and 65.5 feet for SMPDBK, TR66 dimensionless graphs, and HEC-1 respectively. HEC-1 peak depths are from

155 percent to 278 percent of the DAMBRK peak depths. The peak depths for SMPDBK, TR66, and the dimensionless graphs are from 88 percent to 114 percent of the DAMBRK results for the first 40 miles. SMPDBK and the dimensionless graphs peak depths are 138 percent of the DAMBRK depth at mile 50.

68. HEC-1 peak discharges are from 102 to 123 percent of the DAMBRK values. SMPDBK peak discharges are from 93 percent to 141 percent of the DAMBRK values. As discussed below, the TR66 peak discharge was set equal to 100 percent of the DAMBRK value at the dam. The TR66 peak discharge decreased to 67 percent of the DAMBRK value 50 miles below the dam. The dimensionless graphs do not provide peak discharges.

69. Times to peak depth are shown in Table 11. HEC-1 and DAMBRK results varied by six percent or less. SMPDBK times to peak were within 20 percent of DAMBRK. The dimensionless graph times are significantly less than the other models due to the assumption of an instantaneous complete removal of the dam. TR66 does not provide times to peak.

70. The base run breach characteristics consisted of a 500-foot-wide rectangular overtopping breach formed over a 1-hour time period. Breach dimensions varied linearly with time. DAMBRK, HEC-1, and SMPDBK have the capability to model this type of breach. The dimensionless graphs are based on an instantaneous complete removal of the dam. The TR66 breach outflow procedure is based on reservoir depth versus peak breach outflow data from actual past dam failures. This procedure resulted in a peak discharge at the dam of 1,931,000 cfs which is about half the values computed with the other models. The TR66 valley routing procedure is independent of the breach peak discharge computation procedure. Consequently, the DAMBRK peak discharge at the dam of 3,841,000 cfs was adopted for the TR66 analysis shown in the tables with all the other computations following the TR66 procedure.

71. A solution could not be obtained for the base run breach characteristics using FLOW SIM 1 and 2 due to difficulties with computational instability. Successful runs with FLOW SIM 1 and DAMBRK for a breach width of 100 feet had close results. Peak discharges computed with FLOW SIM 1 were from 85 to 90 percent of those computed with DAMBRK. FLOW SIM 1 peak depths were 95 to 98 percent of the DAMBRK peak depths.

Laurel Run Case Study

72. The one peak discharge measurement on Laurel Run was made at a location about a mile below the dam. The peak discharges computed with DAMBRK, FLOW SIM 1, FLOW SIM 2, HEC-1, and SMPDBK are 99, 94, 89, 102, and 145 percent of the measured value.

73. Deviations from the measured high water elevations at seven locations evenly spaced along the channel average 2.1, 2.8, 2.9, 3.4, 4.0, and 9.7 for SMPDBK, FLOW SIM 2, DAMBRK, dimensionless graphs, FLOW SIM 2, and HEC-1, respectively. The average measured flow depth is 15.6 feet. The DAMBRK peak depths are from 85 to 141 percent of the measured values. FLOW SIM 1, FLOW SIM 2, and HEC-1 peak depths are 83 to 159 percent, 78 to 139 percent, and 146 to 198 percent, respectively, of measured values. SMPDBK and the dimensionless graphs peak depths are 90 to 149 percent and 89 to 213 percent of measured.

74. The times to peak depth computed with the alternative models are shown in Table 14. Corresponding measured data are not available. The FLOW SIM 1 and 2 times are not comparable to the DAMBRK times because DAMBRK started the breach at time zero and FLOW SIM 1 and 2 started the breach several minutes after time zero. The instantaneous breach assumption makes the dimensionless graph times less than those computed with the other models. A breach formation time of 15 minutes was used in the other models.

75. The Manning roughness coefficients were increased in the dynamic routing models to prevent supercritical flow from occurring. The values used in DAMBRK and FLOW SIM 1 were twice the actual estimated values. FLOW SIM 2 used roughness coefficient values 1.5 times the actual estimated values. As discussed in the later section on sensitivity analyses, doubling the Manning roughness coefficients raises the peak water surface profile several feet. Thus, the flow depths shown for the three dynamic routing models should be significantly higher than the measured data.

Stillhouse Hollow Case Study

76. The average deviation in peak depth between the values computed by DAMBRK and the other models at seven locations indicated in Table 16 are shown in Table 18. The average deviations for SMPDBK, Military Hydrology Bulletins 9

and 10, TR66, HEC-1, and the dimensionless graph procedure are 11.3 feet, 12.6 feet, 13.5 feet, 15.2 feet and 38.5 feet. The average peak depth computed with DAMBRK was 82.8 feet. Thus, average deviation in peak depth between DAMBRK and SMPDBK is 13.6 percent of the DAMBRK average peak depth. Comparing the dimensionless graphs with DAMBRK, the difference is 66 percent.

77. The HEC-1 peak discharges range from 79 percent to 96 percent of the DAMBRK peak discharges. SMPDBK, TR66, and Military Hydrology Bulletins 9 and 10 peak discharges range from 82 to 96 percent, 91 to 138 percent, 50 to 100 percent, and 63 to 108 percent, respectively, of the corresponding DAMBRK results.

78. The time to peak flow depth computed with HEC-1, SMPDBK, dimensionless graphs, and Bulletins 9 and 10 range from 100 to 130 percent, 93 to 131 percent, 0 to 51 percent, and 0 to 139 percent, of the corresponding DAMBRK values. The dimensionless graph procedure and Military Hydrology Bulletins 9 and 10 are based on the assumption of an instantaneous failure. A breach time of four hours was used in the other models.

79. The peak discharge of the dam was computed to be 1,174,000 cfs using the TR66 procedure which is based on reservoir depth versus discharge data from actual past dam failures. Since the TR66 peak discharge at the dam is much less than the values computed using the other models, the DAMBRK value of 2,783,000 cfs was adopted for the TR66 analysis with all other computations following the TR66 procedure.

Sensitivity Analysis

80. The comparative summary of results presented in the preceding paragraphs was limited to a single base run for each model applied to each case study. Numerous other runs were made with variations in the values of selected input data. Analyses were thus made of the sensitivity of model results to key input parameters. The sensitivity analyses also provided an additional test of model performance by verifying that results respond in a reasonable manner to changes in input data.

81. Various types of sensitivity analyses were conducted. Table 19 is a tabulation showing the types of input data which were varied using different models in the case studies. The results of each of these sensitivity analyses are incorporated in the detailed documentation for each case study. The

results of the sensitivity analyses are illustrated in the other volumes by tabular and graphical profiles of peak discharge, water surface elevation, depth, and time to peak depth. The discussion here is limited to general observations.

82. As previously discussed, the convergence and stability characteristics of the dynamic routing models were also investigated by running the models with alternative values of input parameters. In addition to the types of input data listed in Table 18, extensive analyses directed toward overcoming instability and nonconvergence problems involved altering cross-sectional geometry, distance and time steps, and other parameters.

83. Quantifying breach characteristics is a major area of modeling uncertainty. Accurately mathematically reproducing the actual Teton and Laurel Run breaches is difficult even though information regarding the breaches is available. Predicting the breach characteristics of a dam which has not actually failed is necessarily highly uncertain. Consequently, a number of the sensitivity analyses focused on breach parameters. The effects of varying breach parameters are most pronounced near the dam, diminishing downstream. An analysis of the Teton flood using DAMBRK indicates that changing the breach time from 1 to 2 hours, with all other input data held constant, results in an 8.2-foot decrease in peak stage just below the dam diminishing to a 0.4-foot decrease 9.5 miles below the dam and a few tenths of a foot or less change at locations further downstream. Changing the breach time from 1 hour to 0.5 hour increases the peak stage 1.8 feet at the dam, diminishing to essentially no change 9.5 miles below the dam. A FLOW SIM 1 analysis of the Laurel Run data set indicates that doubling the breach time from 15 to 30 minutes results in a 3.0-foot decrease in peak stage just below the dam and a 0.5-foot decrease 2.5 miles downstream. Reducing the breach time from 15 to 5 minutes results in 2.1-foot peak stage increase just below the dam and a 0.5-foot increase 2.5 miles downstream. A DAMBRK analysis of the prismatic channel case study shows widths of the rectangular breach of 100 feet, 300 feet, and 500 feet result in peak depths of 62.5 feet, 99.2 feet, and 114.6 feet just below the dam. Corresponding depths 50 miles downstream are 42.6 feet, 50.9 feet, and 53.0 feet.

84. The DAMBRK analysis indicates that a 50-percent increase in the Manning n values for Teton results in a 1- to 2-foot increase in peak stage along most of the valley but greater increases, up to almost 15 feet, near the

Table 19

Sensitivity Analyses

Case Study	Model	Input Data Types Included in Sensitivity Analysis
Teton	DAMBRK	Volume losses inactive area delineations Breach time Manning roughness coefficients
Teton	FLOW SIM 1	Breach time Breach side slopes
Teton	HEC-1	NSTPS and NMIN Breach time
Teton	SMPDBK	Cross-section locations Breach width
Prismatic	DAMBRK	Breach width Manning roughness coefficients
Prismatic	HEC-1	NSTPS and NMIN
Laurel Run	DAMBRK	Breach time
Laurel Run	FLOW SIM 1	Simulation starting conditions Breach time
Laurel Run	FLOW SIM 2	Manning roughness coefficients
Laurel Run	HEC-1	Simulation starting conditions NSTPS and NMIN
Laurel Run	SMPDBK	Breach time Manning roughness coefficients
Stillhouse Hollow	DAMBRK	Manning roughness coefficients Breach time and width
Stillhouse Hollow	HEC-1	Breach time and width
Stillhouse Hollow	SMPDBK	Breach time and width

dam. For the prismatic channel case study, DAMBRK indicates that a 50-percent increase in Manning n values result in a 22-foot increase in peak stage at the dam and a 4-foot increase 40 miles below the dam. Doubling Manning n values in the FLOW SIM 2 analysis of Laurel Run increased peak stages from 3 to 6 feet.

85. The Teton Dam failure resulted in a tremendous flood wave inundating a large area of dry ground. American Falls Reservoir, located 100 miles downstream, contained the entire inflow. About a third of the total volume released from Teton Reservoir was lost before reaching American Falls Reservoir. The DAMBRK analysis included runs with and without volume losses. The volume losses have little effect near the dam but decrease the peak stage by up to 10 feet further downstream. This means that near the downstream end of the study reach, peak flow depth not considering the volume loss is over twice the depth with volume loss.

PART V: MODEL EVALUATION

State-of-the-Art

86. DAMBRK, FLOW SIM 1, FLOW SIM 2, HEC-1, SMPDBK, TR66, and the dimensionless graph procedure represent the current state-of-the-art of dam-breach flood forecasting. All of the models were developed within the last 10 years. They provide major improvements over modeling capabilities of a decade ago. A number of significant revisions to the models have occurred within the last 3 or 4 years. The present Corps of Engineers effort to develop an improved tactical dam-breach flood forecasting capability for the Armed Forces is very timely from the perspective of taking advantage of recent advances in the state-of-the-art.

87. Military Hydrology Bulletins 9 and 10 provide a step-by-step manual computation method for developing the outflow hydrograph from a dam breach and routing it downstream. The Defense Intelligence Agency outflow hydrograph procedure improved upon the outflow hydrograph portion of the Military Hydrology Bulletins 9 and 10. These dam-breach flood forecasting procedures were developed during the 1950's and early 1960's and were representative of the state-of-the-art at that time. The computations are rather tedious, involving development of a number of graphs. The outflow hydrograph modeling capability is somewhat limited by the assumption of an instantaneous breach. The Muskingum valley routing method is significantly less accurate than the more recently developed routing models, particularly for a dam-breach flood wave. The Military Hydrology manual procedures have become obsolete with the development of improved methods during the past decade.

88. Dam-breach flood wave modeling capabilities are definitely available to provide meaningful and useful information for practical military and civilian applications. However, model users should be aware that models always have limitations in regard to accuracy. The case study results show a large variation between computed and measured flood wave characteristics and between values computed with the six models evaluated. The accuracy achieved in this study is consistent with that of other similar work reported in the literature. Even with the significantly improved modeling capabilities developed in recent years, dam-breach flood wave modeling is still imprecise.

89. The dam-breach flood wave is a complex phenomenon to model. It is interesting to note that the measured high-water elevations for a reach of the Teton Valley just below the canyon mouth varied by 20 feet between the right and left sides of the valley. The flow was clearly not one-dimensional at this location. However, all of the models are based on the assumption of one-dimensional flow. Even with detailed eye-witness accounts of the breaching of the Teton Dam, the breach is difficult to model accurately. In the case of both Teton and Laurel Run, the breach did not form instantaneously nor did the breach dimensions vary linearly with time. Numerous factors determine model accuracy. The significance of each factor depends upon the particular dam and application.

90. In general, the accuracy achieved in modeling flows in rivers and reservoirs and similar types of modeling is highly dependent upon how well the model is calibrated. Parameters are adjusted until the model reproduces results known to be correct from actual observation. However, field measurements of the characteristics of an actual flood wave similar to that being modeled must be available if a flood routing model is to be calibrated. Since a dam-breach flood will usually be much larger than the flood of record for a river, calibration of a dam-breach flood wave model is difficult. The case study results reported herein were computed with uncalibrated input data. No attempt was made in this study to adjust input parameters to improve the results.

Outflow Hydrograph Computations

91. The case study analyses performed with DAMBRK. FLOW SIM 1 and 2, and HEC-1 involved computing the breach outflow hydrograph using storage routing and a breach simulation algorithm in which breach dimensions grow linearly with time. The four models have comparable reservoir storage routing and breach simulation algorithms.

92. The dimensionless graphs procedure is based on the assumption of an instantaneous complete removal of the dam. The DIA outflow hydrograph procedure assumes an instantaneous partial breach. The instantaneous breach assumption is a significant limitation for the models.

93. TR66 involves computation of a peak breach outflow from a reservoir depth versus peak outflow relationship based on data from actual past dam

failures. This relationship produced results significantly different from the other models in the case studies. Since military applications will involve intentionally caused dam breaches significantly different from the actual past dam failures used to develop the TR66 relationship, the usefulness of this method for military purposes is limited. However, the TR66 technique for computing the peak breach outflow is independent of the remainder of the computations. Therefore, the TR66 valley routing can be performed with a peak breach discharge determined by another method.

94. SMPDBK provides for a rectangular breach with dimensions increasing linearly with time. SMPDBK computes the peak discharge, not the entire breach outflow hydrograph.

95. DAMBRK and FLOW SIM 1 and 2 also have options for dynamic reservoir routing, but these options were not applied in the case studies. In most cases, dynamic reservoir routing is not expected to be advantageous over storage routing. However, dynamic routing should be more accurate than storage routing for a long, narrow reservoir with a significantly sloping water surface.

96. FLOW SIM 1 and 2 have a breach simulation option in which the rate of growth of the breach is determined with an erosion formula. An erosion breach simulation model has also recently been developed for use with DAMBRK (Fread, 1984). The breach simulation algorithms based on erosion formulas were not investigated by this study.

Valley Routing

97. The key difference between the dam-breach flood wave models is the method used for routing the hydrograph through the valley below the dam. The models can be divided into three categories based on valley routing techniques as follows:

- a. Dynamic routing models (DAMBRK, FLOW SIM 1 and 2).
- b. Simplified dynamic routing models (SMPDBK and dimensionless graph procedure).
- c. Nondynamic routing models (HEC-1, TR66, Military Hydrology Bulletins 9 and 10).

Dynamic routing models are based on a numerical solution of the St. Venant equations. The dynamic routing models are generally the most accurate but

also most difficult to use of the models. The other two categories consist of models which are simpler to use but also less accurate. The simplified dynamic routing models are based on generalized relationships between selected input parameters and selected routing output quantities which were predeveloped using a dynamic routing model. The third category consists of models which use techniques other than dynamic routing.

Dynamic routing models

98. A dynamic routing model should be used for military or civilian applications whenever obtaining a maximum practical level of accuracy is important and adequate manpower, time, and computer resources are available. Although dynamic routing is based on simplifying assumptions including one-dimensional flow, it is the most theoretically correct of the state-of-the-art routing techniques. The case study analyses confirmed that the dynamic routing models are the most versatile and accurate of the models tested.

99. DAMBRK, FLOW SIM 1, and FLOW SIM 2 are the three dynamic wave models investigated. FLOW SIM 1 and 2 use identically the same input data. The numerical solution technique used in the dynamic routing is the only difference between these two models. FLOW SIM 1 has an explicit solution and FLOW SIM 2 has an implicit finite difference solution of the St. Venant equations. DAMBRK has an implicit four-point finite difference solution of the St. Venant equations similar to that contained in FLOW SIM 2.

100. The dynamic routing models can reflect a significantly broader range of conditions, such as backwater effects and inactive versus active flow areas, than the other models. The dynamic routing models are generally more accurate than the other models. However, nonconvergence and computational instability problems may require significant modification of input data to obtain solutions. Smoothing the valley geometry and modifying other input data can significantly reduce the accuracy of the results. Computational problems also make the dynamic routing models much more complicated to use than the other models.

101. Results obtained with DAMBRK, FLOW SIM 1 and FLOW SIM 2 were found to be very close in the case studies whenever solutions were obtained with comparable input data. The results are too close to conclude that one of the three models is more or less accurate than the others. FLOW SIM 1 and 2 in particular yield comparable results for the same input data, if solutions are obtained. The primary factor in differentiating between the models is how

much the input data had to be modified to obtain a solution. Stated another way, model performance is measured in terms of the range of input data values for which a solution is obtained versus a termination of the calculations due to nonconvergence or instability. DAMBRK performed significantly better in the case studies than FLOW SIM 1 and 2 in this regard. FLOW SIM 1 performed better than FLOW SIM 2.

Simplified dynamic routing models

102. A simplified dam-breach flood wave model is needed for military applications in which a mainframe computer is not available and/or manpower or time is limited. SMPDBK and the dimensionless graph procedure are extremely easy to use compared with the dynamic routing models. The results of the simplified dynamic routing models were reasonably close to the dynamic routing models in the case studies.

103. SMPDBK was somewhat more accurate in the case studies than the dimensionless graphs procedure. With a microcomputer, SMPDBK is quicker to use than the dimensionless graphs procedure which is done with manual computations. The dimensionless graph procedure is a little easier to use than the manual version of SMPDBK.

104. The prismatic channel assumption is a significant limitation of both models which is particularly evident in the Teton case study. The assumption of an instantaneous complete removal of the dam limits the accuracy of the dimensionless graphs, particularly near the dam, in the general case in which the assumption is not valid.

105. SMPDBK was generally more accurate than HEC-1 and TR66 in the case studies. SMPDBK is much easier to use than HEC-1 and TR66.

Nondynamic routing models

106. Computational instability was not a problem with HEC-1 in the case studies. Although warnings that the modified Puls routing may be numerically unstable for certain ranges of outflow were often obtained, the computed hydrographs were generally reasonable. Consequently, HEC-1 was found to be much simpler to use than DAMBRK and FLOW SIM 1 and 2. The peak discharges and times to peak computed with HEC-1 were reasonably close to the measured data and DAMBRK results. However, the peak depths were highly inaccurate. The program option was used in which the Mannings equation and an assumption of uniform flow is used to compute the outflow versus storage functions and the discharge versus stage functions. Although not investigated, the results

could be somewhat improved by developing outflow versus storage functions with HEC-2 to be furnished as input data to HEC-1. This would more than double the effort required to use HEC-1.

107. The case study peak water surface elevation profiles computed with HEC-1 are less accurate than those computed with SMPDBK. HEC-1 performed as well or better than SMPDBK in regard to peak discharges and time to peak stage. Also, HEC-1 provides an entire hydrograph while SMPDBK is limited to peak discharge. However, peak water surface elevation is the most important model result in most applications.

108. The manual TR66 procedure is time-consuming due primarily to the requirement for developing stage versus discharge and storage versus discharge relationships for the valley routing. The same relationships are required for HEC-1 but are developed by the model from inputted cross-section data. For both models, these relationships were developed based on Mannings equation and the assumption of uniform flow. The manual computations makes TR66 much more difficult to use than HEC-1.

109. The HEC-1 computer program used in this study requires a mainframe computer. However, the Hydrologic Engineering Center has recently developed a microcomputer version of the program. The HEC-1 program is actually a package of various types of computations. The computational options available for dam breach flood wave modeling are a relatively small portion of the total package of options.

PART VI: CONCLUSIONS

110. The case study analyses provided a convenient basis for evaluating and comparing six dam-breach flood wave models. The quantitative results summarized in this report should be useful to researchers and practitioners interested in developing an in-depth understanding of the performance of the models under various conditions. Key general conclusions derived from the study are provided in the following paragraphs.

111. Although modeling capabilities are available to provide meaningful and useful information for practical military application, dam-breach flood wave modeling is not highly precise. Model users should be aware of limitations in accuracy and preciseness.

112. Although the dynamic wave models are significantly more accurate and versatile than the other models, computational instability and nonconvergence problems are significant concerns in their application. Training and experience in numerical computer modeling are needed to use the dynamic routing models.

113. A dynamic wave model should be used whenever obtaining maximum practical accuracy is important and adequate manpower, time, and computer resources are available. A simpler model is needed for obtaining solutions expeditiously with limited resources.

114. The National Weather Service DAMBRK and SMPDBK are the optimal models for immediate adoption by the military for tactical forecast applications.

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